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UNIAXIAL STRESS-STRAIN BEHAVIOR OF UNSATURATED SOILS AT HIGH STRAIN RATES

Dr. George E. Veyera

WL/FIVC
Wright Laboratory Flight Dynamics Directorate
Air Force Material Command
Tyndall AFB FL 32403-5323

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NOTICES

This report has been reviewed and is approved for publication.

Richard A. Reid

RICHARD A. REID, Capt, USAF
Project Officer

William S. Strickland

WILLIAM S. STRICKLAND, GM-14
Chief, Engineering Research Division

Jon B. Anderson

JON B. ANDERSON
Chief, Air Base Survivability Branch

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Dr. George E Veyera, Associate Profesor
Department of Civil and Environmental Engineering
University of Rhode Island

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13. ABSTRACT (Maximum 200 words)
The Split-Hopkinson Pressure Bar was used to study the uniaxial stress-strain behavior of compacted moist soils under one-dimensional, undrained, confined compression loading at high strain rates (1000/sec and 2000/sec. Three soils, Eglin Sand, Tyndall sand and Ottawa 20-20 sand were tested. The results suggest that the stress-strain response is dominated by the water-phase from the lock-up strain and beyond. The soil skeleton dominates the response from the start of loading up to the lock-up strain. It appears that there may be some strain-rate effects, however, the data are insufficient to adequately demonstrate this and further investigation is suggested.
The research described in this report has demonstrated that the Split-Hopkinson Bar System is a viable technique for high strain rate dynamic geotechnical testing of of unsaturated, saturated, and dry soils, and provides a framework for conducting further studies.

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PREFACE

This report was originally prepared by Universal Energy Systems, Inc, under Contract number F49620-[illegible]-C-[illegible]13 for the Air Force Office of Scientific Research, Bolling Air Force Base, Washington D.C. It contains results of Summer Faculty Research sponsored by Air Force Office of Scientific Research and published by the Civil Engineering Research Division, of Research, Development and Acquisition Directorate, Air Force Civil Engineering Support Agency, 139 Barnes Drive, Tyndall Air Force Base, Florida 32403-5323.

This report covers research performed by Dr. George E. Veyera of the University of Rhode Island between June and August 1992. The report is being reprinted and submitted to Defense Technical Information center because of its widespread interest to the DOD Engineering and Services community. The RACS Mentor was Dr. Allen Ross. The AL/EQ Summer Faculty Coordinator was Mary E. Reynolds.

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Symbol	Definition
c_o	wave propagation velocity of the incident and transmitter bars
C_c	coefficient of curvature
C_u	coefficient of uniformity
dt	denotes time derivative
D	particle diameter
D_{50}	particle diameter for which 50% is finer than
E	modulus of elasticity (Youngs modulus)
L_o	initial specimen length
n	number of points used in FFT smoothing algorithm
S	degree of saturation
t	time
ε	axial compressive strain
ε_i	incident axial compressive strain
ε_r	reflected axial compressive strain
ε_s	average specimen axial compressive strain
$\dot{\varepsilon}_s$	average specimen axial compressive strain rate
ε_t	transmitted axial compressive strain
σ_s	average specimen axial compressive stress

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SECTION I

INTRODUCTION

Current empirical relationships for predicting soil pressure as a function of standoff distance from a buried explosive typically have a variation of ± 50% or more and use material properties data based on conventional weapons effects in dry soils. However, most soils, whether naturally deposited or placed as select engineered fill, exist with moisture at saturations somewhere between 0% and 100%. The reaction of a structure to a specified loading can usually be determined, however, there are no theoretical, numerical or empirical methods available for predicting the groundshock energy arriving at a structure in unsaturated soils. This arises from the fact that dynamic load transfer mechanisms in soils are not well defined especially when moisture is present. In addition, there is little if any actual data available for the transient dynamic behavior of unsaturated soils, particularly at high strain rates.

The ability of a soil to transmit applied dynamic stresses (energy) is of particular interest to the U.S. Air Force with respect to military protective construction and survivability designs. Typical engineering analyses assume that little or no material property changes occur under dynamic loadings and in addition, analyses do not account for the effects of saturation (moisture conditions) on the stress-strain behavior of soils. This is primarily due to an incomplete understanding of soil behavior under transient loadings and uncertainties about field boundary conditions. Results from U.S Air Force field and laboratory tests with explosive detonations in soils have shown that material property changes do in fact occur and that variations in soil stiffness (or compressibility) significantly affect both dynamic and static stress behavior. The research described in this report is important to the U.S. Air Force since there are currently no theoretical, empirical or numerical methods available for predicting the dynamic uniaxial stress-strain response of unsaturated soils from loading environments such as those produced by conventional weapons effects.

Recent research (2, 3, 9, 25-29) using the SHPB facility at AFCESA/RACS to study unsaturated soil behavior has shown that: (a) the presence and amount of moisture significantly affects the dynamic and static response of soil specimens; and (b) the amount of stress transmitted, stiffness, wave speed and compressibility in unsaturated soils varies with the amount of moisture present during compaction. Experimental evidence from a number of researchers suggests that such behavior for both dynamic and static loading conditions can be attributed to variations in soil compressibility and soil microstructure as a result of conditions during compaction including the compaction method used and the amount of moisture present during compaction (2, 3, 5, 9, 15, 16, 18-23, 25-29, 32). While the effects moisture on soil behavior as described above have been observed experimentally, a clear and concise explanation of the phenomenon is not currently available. This is primarily due to the fact that the multiphase behavior of unsaturated soils, the interaction between the individual phases (air, water and solid), and the mechanics of load transfer mechanisms in soils are not well understood.

The research described herein was performed as a part of the 1992 Summer Faculty Research Program (SFRP) to investigate the undrained behavior of unsaturated soils subjected to dynamic confined uniaxial compression loading at high strain rates using the Split-Hopkinson Pressure Bar (SHPB) at AFCESA/RACS. The SHPB device can be used to examine the influence of selected parameters on the dynamic response of many engineering materials including soils. With reference to this research, the term "dynamic" defines large amplitude, high strain rate loadings as opposed to the low strain oscillatory frequency pulses used in wave attenuation studies. The results of studies such as that described herein will lead to a better fundamental understanding of the load transfer mechanisms and constitutive relationships for unsaturated soils and have direct applications to groundshock prediction techniques including stress transmission to structures.

A. RESEARCH OBJECTIVES

The primary objective of the summer research was to study the uniaxial stress-strain behavior of compacted moist soils under one-dimensional, undrained, confined compression loading at high strain rates using the SHPB at AFCESA/RACS. The SHPB testing system has been successfully used to evaluate metals, concrete, composites and foams at high rates of strain and the work described herein included the development of special equipment and techniques for using the SHPB with soils for which limited testing has been done. Particular emphasis was on examining the influence of saturation and strain rate on dynamic soil behavior. The research described is important to the U.S. Air Force since there are currently no theoretical, empirical or numerical methods available for predicting the dynamic stress-strain response of unsaturated soils to conventional weapons loading environments and such information is needed for more rational military protective construction and survivability designs.

B. BACKGROUND

Differences in the stress-strain response for dry and moist soils under both dynamic and static one-dimensional loading conditions have been observed experimentally by a number of researchers at various strain rates. Farr and Woods (6) accurately describe the current state of affairs with regards to the uniaxial stress-strain behavior of soils: "It has long been recognized that the one-dimensional or uniaxial strain response of most soils subjected to high intensity transient loads differs from the response measured under static conditions. As the time to peak pressure decreases, most soils exhibit a stiffening of the loading stress-strain response. The stiffening is usually referred to as a time or loading rate effect. Some researchers (14, 30, 31) have suggested that, as the time to peak pressure approaches the submillisecond range, a drastic increase (up to tenfold) in the loading constrained modulus occurs for partially saturated granular soils under unconsolidated-undrained conditions. The existence of this effect has been the subject of much debate." A number of researchers have attempted to address the controversy surrounding the issue of strain rate effects in both dry and unsaturated soils.

Hendron (12) conducted one-dimensional confined compression tests on dry sands at very low loading rates, noting that the shape of the stress-strain curves became more S-shaped with increasing stress levels. Whitman et al. (31) also observed the typical S-shaped curve for dry sands and propose the following order of events with increasing applied load to describe this behavior: initially, deformations occur at the grain contact points as the individual grains deform; this is followed by a decrease in resistance to straining as the grains slide relative to each other; finally, an increase in resistance to straining occurs as the grains rearrange into a denser packing. Such behavior has been demonstrated theoretically from an analysis of a regular array of elastic spheres and has been shown to be dependent on the confinement boundary conditions (24). For a condition of zero lateral strain, the stress-strain curve exhibits a "strain hardening" effect with increasing stress (curve is concave towards the stress axis). In addition, Whitman et al. (31) noted an increase in modulus with decreasing time to peak loading for rise times ranging from the millisecond to seconds range attributed the stiffening response to strain rate effects.

High pressure static uniaxial confined compression tests were conducted on moist specimens of sandy silt at various dry densities by Hendron et al. (13). They observed increases in stiffness with increasing saturation (Figure 1) and from their results, concluded that the key variables in uniaxial stress-strain behavior are void ratio and saturation. Wu et al. (32) tested moist silty soils at small strains in the resonant column device and found a significant increase in the dynamic shearing modulus for specimens compacted moist at saturations in the range of from 5 to 20% (Figure 2). Whitman (30) indicated that rate effects become very important at submillisecond loading times and theorized that at high saturations, the pore phase of the soil is much stiffer than the soil skeleton (pore fluid compressibility

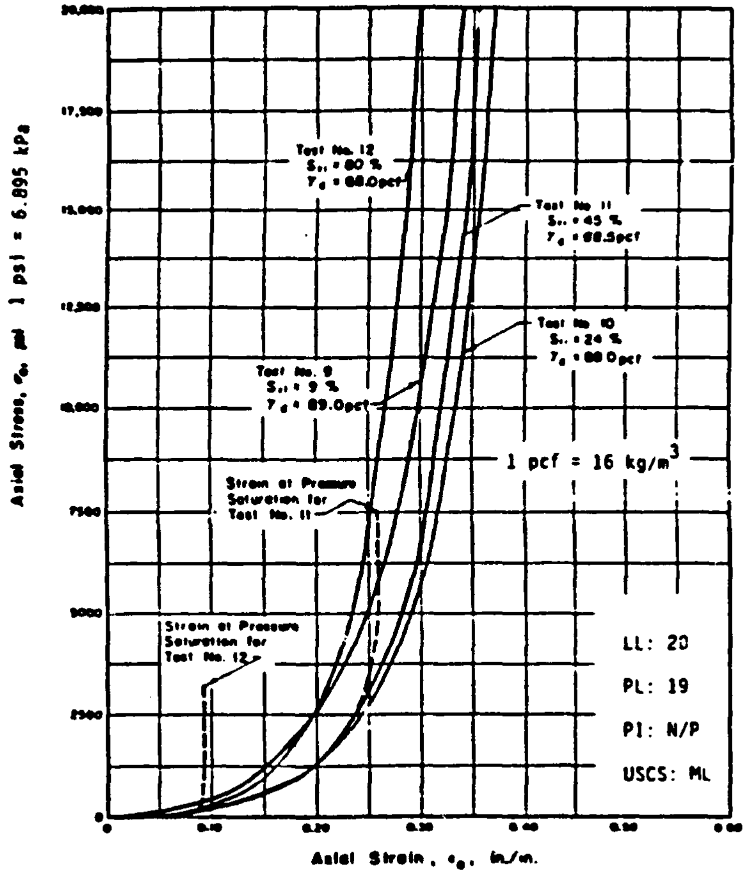

FIGURE 1. Uniaxial Compression Stress-Strain Response for Sandy Silt as a Function of Saturation (Hendron et al. (13)).

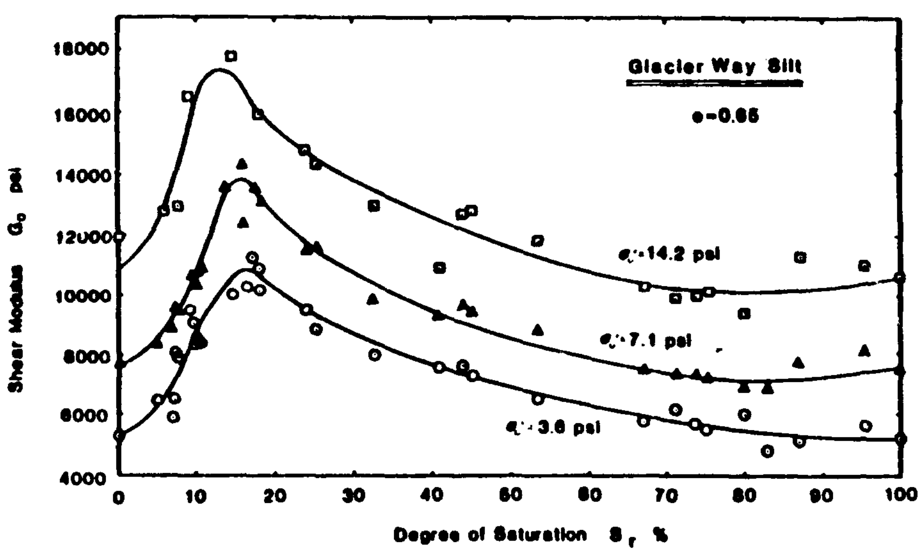

FIGURE 2. Low Amplitude Shear Modulus as a Function of Saturation for Glacier Way Silt (Wu et al., (32)).

dominates), while at lower saturations the skeleton matrix is stiffer than the pore phase (skeleton compressibility dominates). Jackson et al. (14) conducted unconsolidated-undrained one-dimensional confined compression tests on several different air dry soils at various loading rates and observed loading rate effects at submillisecond loading times, with the constrained modulus increasing by an order of magnitude in going from 0.1 to 1.0 ms rise time to peak stress. Farr and Woods (6) used a modified version of Jackson et al.'s (14) experimental apparatus to conduct similar tests on moist carbonate sand. They observed a progressive stiffening in the stress-strain response with faster loading times to peak stress and noted rate dependent effects occurred even at multi-second loading rates for the carbonate sand.

Limited research has also been conducted using the Split-Hopkinson Pressure Bar to study the uniaxial stress-strain behavior of compacted moist soil under one-dimensional confined compression loading at high strain rates (7, 8, 11). The results of these studies indicated that the uniaxial stress-strain response is primarily governed by the initial gas filled porosity of the soil specimen, and that strain-rate effects did not occur at strains less than the initial gas filled porosity. However, it should be noted that the specimen container boundary conditions were not for completely undrained conditions and therefore, this may have had significant effects on the results. The SHPB at AFESC/RACS (9, 25-29) has also been used to study stress transmission characteristics of unsaturated soils for conditions of undrained confined uniaxial compression. The AFCESA/RACS facilities and equipment were used in the research described in this report to examine the undrained uniaxial compressive behavior of unsaturated soils at high strain rates. Detailed information about the SHPB device at AFCESA/RACS and results of recent investigations to study dynamic soil behavior can be found in the references cited above.

SECTION II

EXPERIMENTAL INVESTIGATION

A. DESCRIPTION OF SOILS TESTED

Three different granular soils were tested using the AFCESA/RACS SHPB facility: Eglin sand (from Eglin AFB), Tyndall sand (from Tyndall AFB), and Ottawa 20-30 sand (commercially available from the Ottawa Silica Company). Representative soil samples were randomly obtained for analysis and testing from bulk quantities of each material using standard sample splitting procedures. The Eglin sand is a medium to fine, angular to subangular sand with about 7% fines; the Tyndall sand is a fine, uniform, subangular sand with no fines; and the Ottawa 20-30 sand is a uniformly graded, subrounded to rounded, medium sand with no fines. Various physical index properties data were obtained for each sand and the results are summarized in Table 1. A comparison of the grain size distributions is shown on Figure 3.

B. PREPARATION OF COMPACTED SOIL SPECIMENS

Specimens of the Ottawa 20-30 sand were dynamically compacted to a constant dry density at varying degrees of saturation (different initial moisture contents) in a 7.62 cm (6.00 inches) long, 2.54 cm (1.00 inch) thick seamless stainless steel container which had an inside diameter of 5.08 cm (2.00 inches). The thick-walled stainless steel tube was used to simulate the one-dimensional, confined uniaxial loading condition typically encountered near explosive detonations in the field. Soil specimens are held in the container by two 0.635 cm (0.250 inch) thick stainless steel wafers fitted with o-ring seals used to prevent drainage of pore fluid during compaction and testing. One wafer is inserted prior to compaction and the other one after

TABLE 1. Physical Properties of Eglin, Tyndall and Ottawa 20-30 Sands.

	Eglin	Tyndall	Ottawa 20-30
[a]USCS Classification	SP-SM	SP	SP
Specific Gravity	2.65	2.65	2.65
D_{50} Particle Size (mm)	0.26	0.19	[illegible]
[b]C_u	3.41	1.18	[illegible]
[c]C_c	1.29	0.95	1.03
[d]Percent passing #100 sieve (%)	12	2	<1
[d]Percent passing #200 sieve (%)	7	0	0
[e]Maximum dry density (kg/m^3)	1,670	1,630	1,720
[f]Minimum dry density (kg/m^3)	1,450	1,450	1,560
Maximum void ratio	0.817	0.817	0.705
Minimum void ratio	0.590	0.621	0.545

Note:
[a]Unified Soil Classification System (1)
[b]Coefficient of Uniformity
[c]Coefficient of Curvature
[d]U.S. Standard Sieve
[e]ASTM D4253 (1)
[f]ASTM D4254 (1)

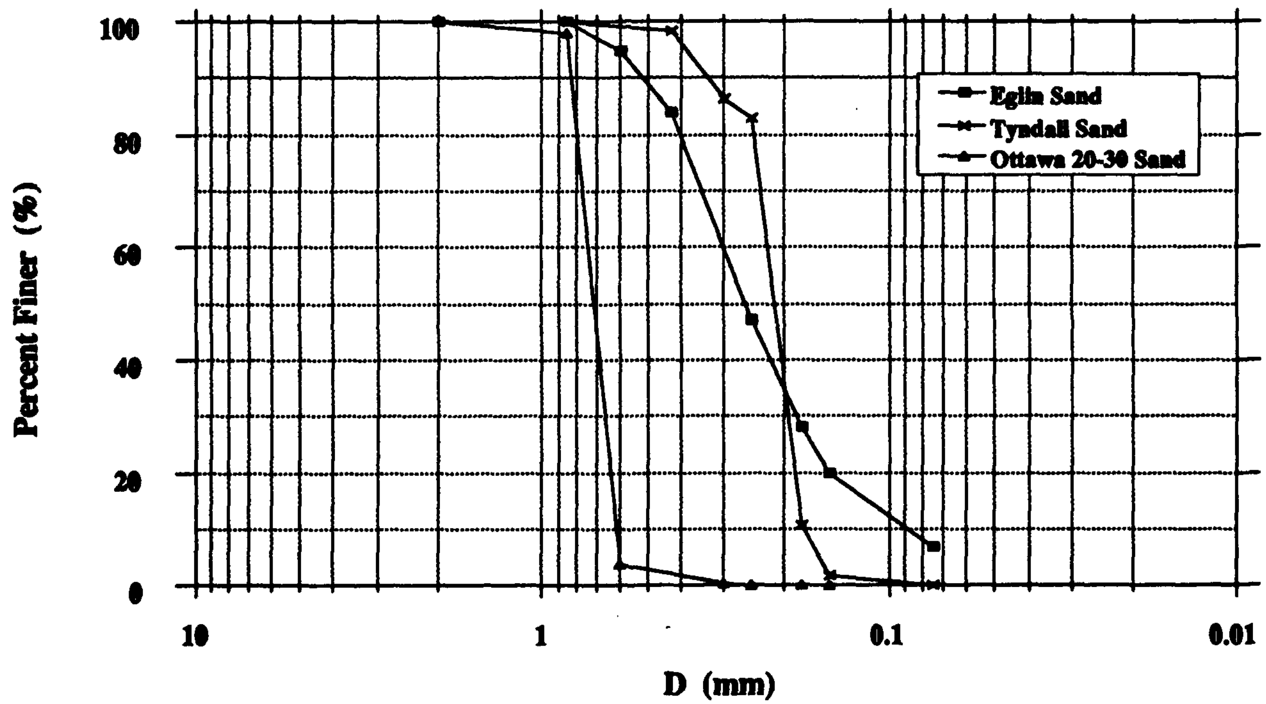

FIGURE 3. Grain Size Distributions for Eglin, Tyndall and Ottawa 20-30 Sands.

compaction is completed. Special care is taken when placing the second wafer after compaction to ensure full contact with the specimen. Figure 4 shows a compacted specimen prepared for testing in the SHPB.

A Standard Proctor hammer, ASTM D-698 (1), was used to consistently apply a controlled amount of compactive effort per impact to each soil specimen (7.5 Joules or 5.5 ft-lbs per impact). All test specimens were formed using a single individually compacted layer such that a final specimen length of 1.27 cm (0.50 inches) or 0.635 cm (0.25 inches) would be obtained at the maximum dry density for each soil. Saturations were varied from 0% to 100% for each soil.

In preparing moist specimens, the required amount of water for a given degree of saturation (at final compacted density) was added to the originally dry soil, thoroughly mixed in and then allowed to equilibrate before compacting. Although the dry density was constant for each specimen, the amount of compactive effort required varied with the amount of moisture (saturation). For specimens ranging from 0% to about 80% saturation, the tests were conducted on unsaturated specimens which implies that both continuous air and water phases exist in the soil (e.g., there are no isolated air or water pockets in the grain matrix). For most soils, this generally occurs at saturations less than about 85% (4). Fully saturated specimens (no air), were also prepared. For dry specimens, the dry soil was poured directly into the tube and then compacted.

C. SPLIT-HOPKINSON PRESSURE BAR

Figure 5 shows an overview of the AFCESA/RACS Split-Hopkinson Pressure Bar testing facility which consists of several separate but intimately related components: (a) a dynamic loading system which includes a nitrogen pressurized cannon used to fire 5.08 cm (2.00 inch) diameter stainless steel projectiles (striker) of varying lengths at the incident bar; (b) a stainless steel incident bar 5.08 cm (2.00 inch) in diameter and 3.66 m (12 feet) in length; (c)

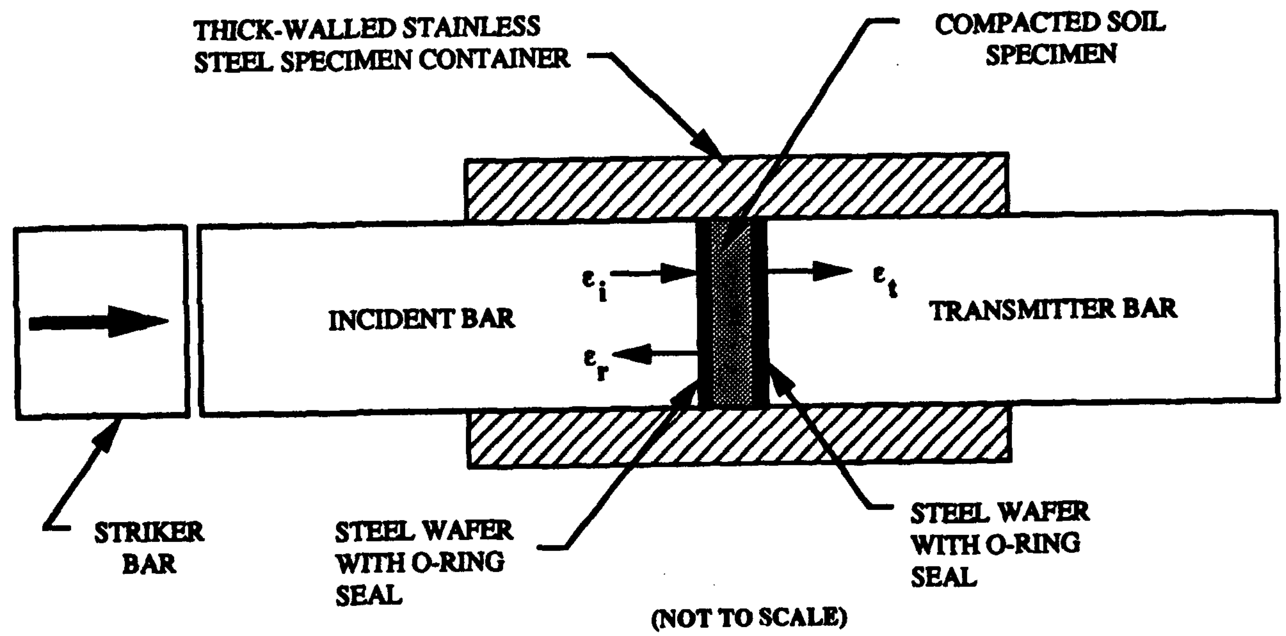

FIGURE 4. Schematic of Compacted Soil Specimen in the Split-Hopkinson Pressure Bar.

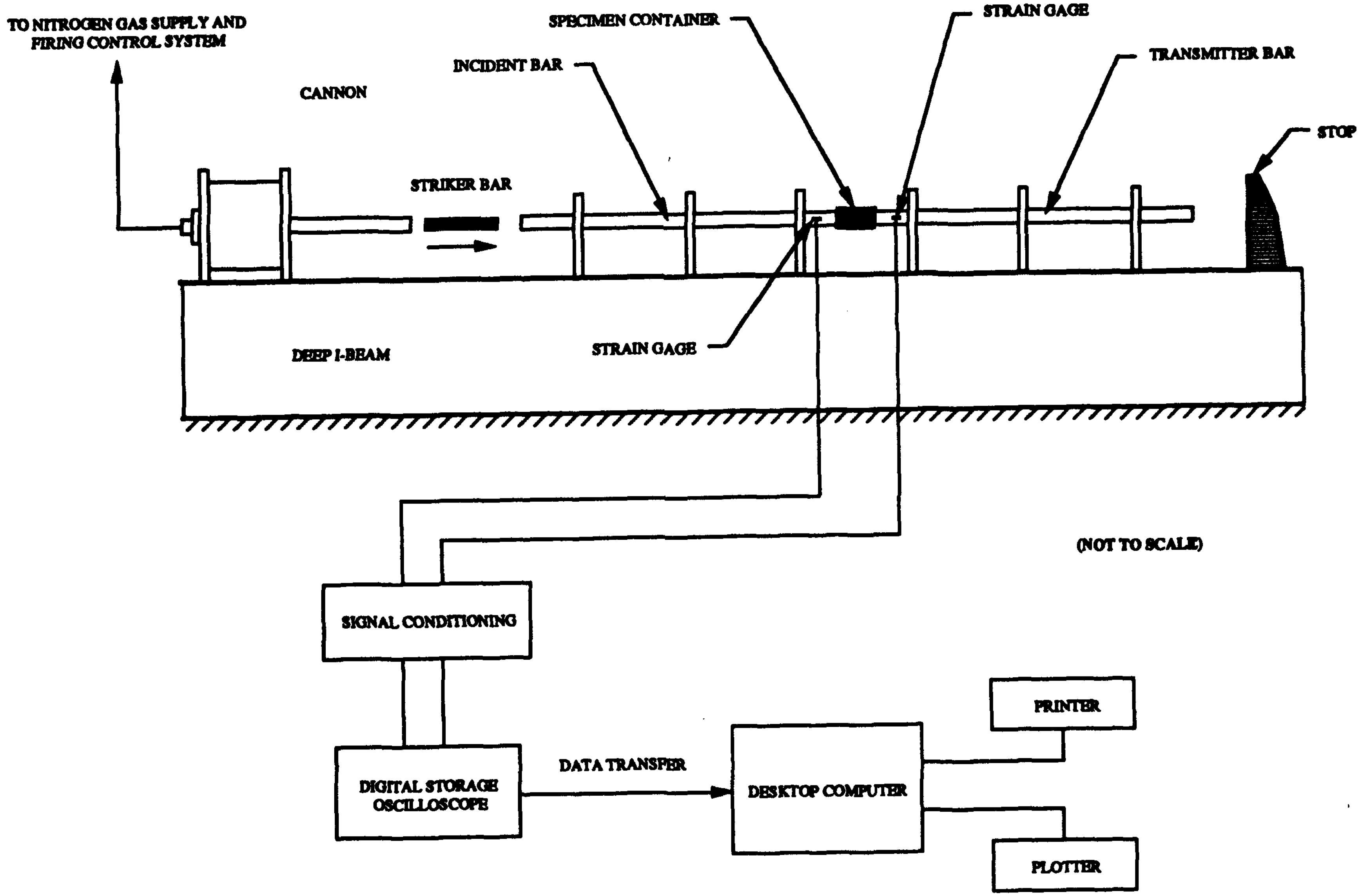

FIGURE 5. Schematic of Split-Hopkinson Pressure Bar Test Facility.

a stainless steel transmitter bar 5.08 cm (2.00 inch) in diameter and 3.35 m (11 feet) in length; (d) electronic strain gage instrumentation with power supplies and amplifiers; (e) a digital storage oscilloscope for data acquisition; and (f) a desktop computer for data reduction and analysis.

A series of tests was conducted on the Eglin, Tyndall and Ottawa 20-30 sands at varying degrees of saturation between 0% and 100% and at two different strain rates (Tyndall sand was only tested at one strain rate - 1000/sec). Strain rates were approximately 1000/sec and 2000/sec which were obtained by testing compacted specimens of two different lengths: 1.27 cm (0.50 inches) or 0.635 cm (0.25 inches), respectively. In all tests, a 0.653 m (25.7 inch) long striker was fired at a cannon pressure of 690 kPa (100 psi), which produced a square wave input stress of approximately 225 MPa (37,000 psi) with a rise time to peak stress on the order of 50 microseconds and a 257 microsecond pulse width. The same cannon pressure, striker bar length and specimen container were used in all tests for each sand.

SECTION III

RESULTS AND DISCUSSION

The SHPB system provides measurements of the incident, reflected and transmitted strains in the incident and transmitter bars through electronic strain gage instrumentation and therefore, direct measurements of stress and strain within the specimen itself are not made (Figures 4 and 5). The average strain, average strain rate and average stress in the SHPB specimen can be determined from the strain gage data using the following relationships derived from elastic theory for one-dimensional stress wave propagation in a rod (17, 25):

Average specimen strain: $$\varepsilon_s = -\frac{2c_o}{L_o}\int_0^t \varepsilon_r \, dt \qquad \text{(Eq. 1)}$$

Average specimen strain rate: $$\dot{\varepsilon}_s = -\frac{2c_o}{L_o}\varepsilon_r \qquad \text{(Eq. 2)}$$

Average specimen stress: $$\sigma_s = E\varepsilon_t \qquad \text{(Eq. 3)}$$

where: L_o is the initial specimen length, c_o is the wave propagation velocity of the incident and transmitter bars, E is Young's modulus, and ε_i , ε_r and ε_t are the incident, reflected and transmitted strains, respectively.

The derivation of Eqns. 1, 2 and 3, assumes: a) the incident and transmitter bars are of same material (i.e., they have the same wave speed); b) loading stresses are in the elastic range of the bars; c) a uniform one-dimensional stress state is developed in the specimen; d) the forces on each end of the specimen are equal; and e) the cross-sectional areas of the bars and the specimen are equal. These equations are used in analyzing the raw data to develop the uniaxial stress-strain curves (Figures 6 and 7). The data analysis includes a dispersion correction to account for wave spreading in the bars and an FFT using 17 point smoothing (n=17).

Figures 8, 9 and 10 show the uniaxial stress-strain results for the Eglin, Tyndall and Ottawa 20-30 sands, respectively. Data were obtained for the Eglin and Ottawa 20-30 sands at strain rates of 1000/sec and 2000/sec, while the Tyndall sand data were only obtained at a strain rate of 1000/sec. In addition, there were some difficulties in obtaining reliable data for the Ottawa 20-30 sand at 100% saturation, therefore those data have not been included. There are some general features observable in the stress-strain data: a) an initially steep loading portion of the curves which is probably associated with the initial rise in the loading pulse and appears to be strain rate independent; b) the slopes of each curve are about the same after the initial steep portion up to the lock-up strain; c) the initial saturation affects the point at which lock-up occurs (the higher the initial saturation, the smaller the strain required); and d) after lock-up the curves exhibit a stiffening response with a slope approximately that of pure water (Figure 11). In comparing the test results at different strain rates, it appears that there may be some strain rate effects, however, the data are insufficient to adequately demonstrate this and further investigations are required. The data at a strain rate of 2000/sec show increasing dispersion (waviness) in the curves for the Eglin sand, being much less for the Ottawa 20-30 sand. The dispersion effect is most likely due to the significant differences in particle characteristics of each soil (grain size, shape and size distribution), and it may also be that the small specimen length is approaching some limiting value in terms of particle size relative to loading pulse length.

In comparing the three different soils together it can be seen that: a) the slopes of the curves after the initial steep portion are nearly identical regardless of the initial saturation and soil type; b) the initiation of the lock-up strain is somewhat different for each soil at the same saturation; c) lock-up was not developed in the Ottawa 20-30 sand below 80% saturation.

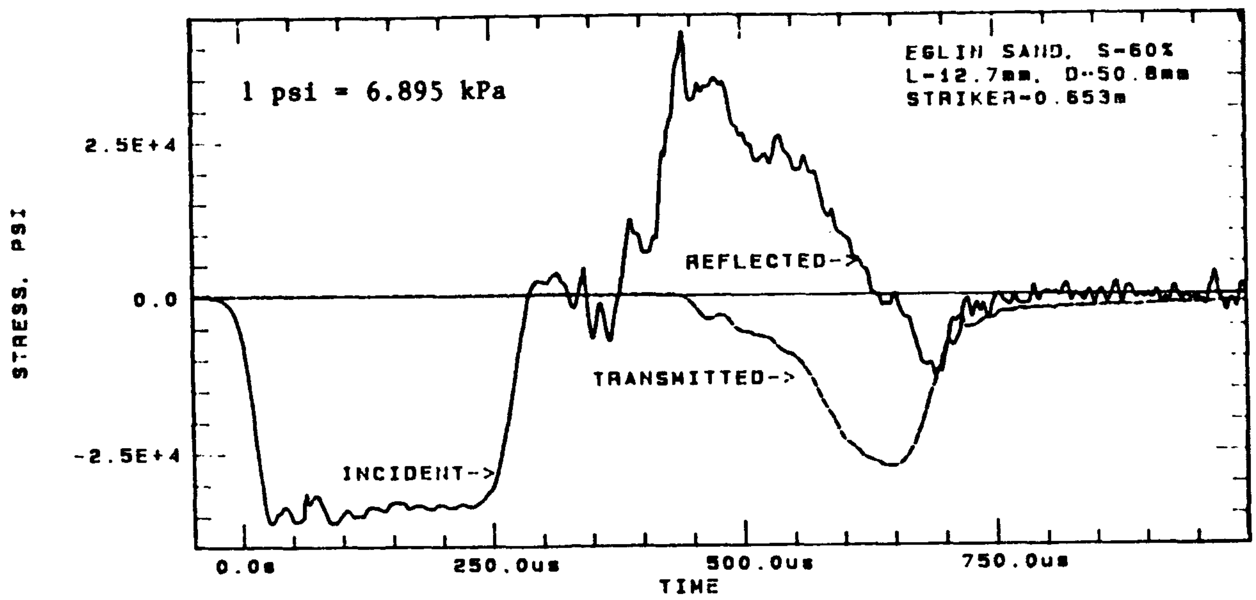

FIGURE 6. Typical SHPB Data for Incident, Reflected and Transmitted Stresses as a Function of Time for Uniaxial Compressive Loading.

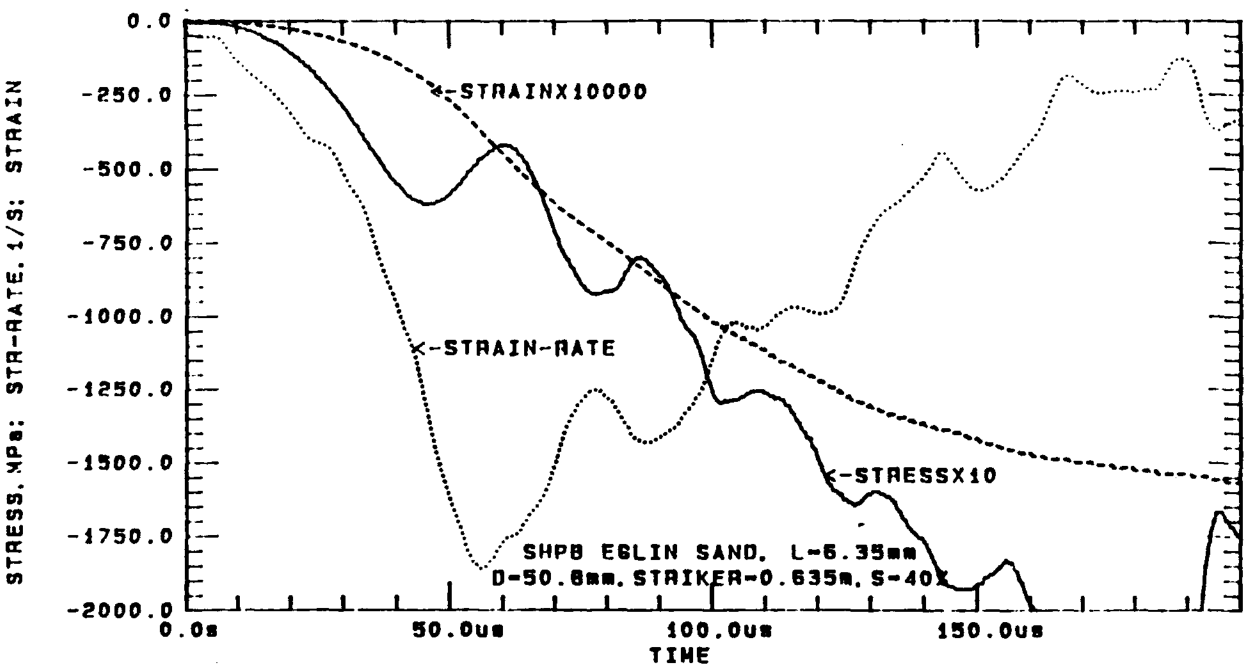

FIGURE 7. Typical SHPB Data for Strain, Strain Rate and Stress as a Function of Time for Uniaxial Compressive Loading.

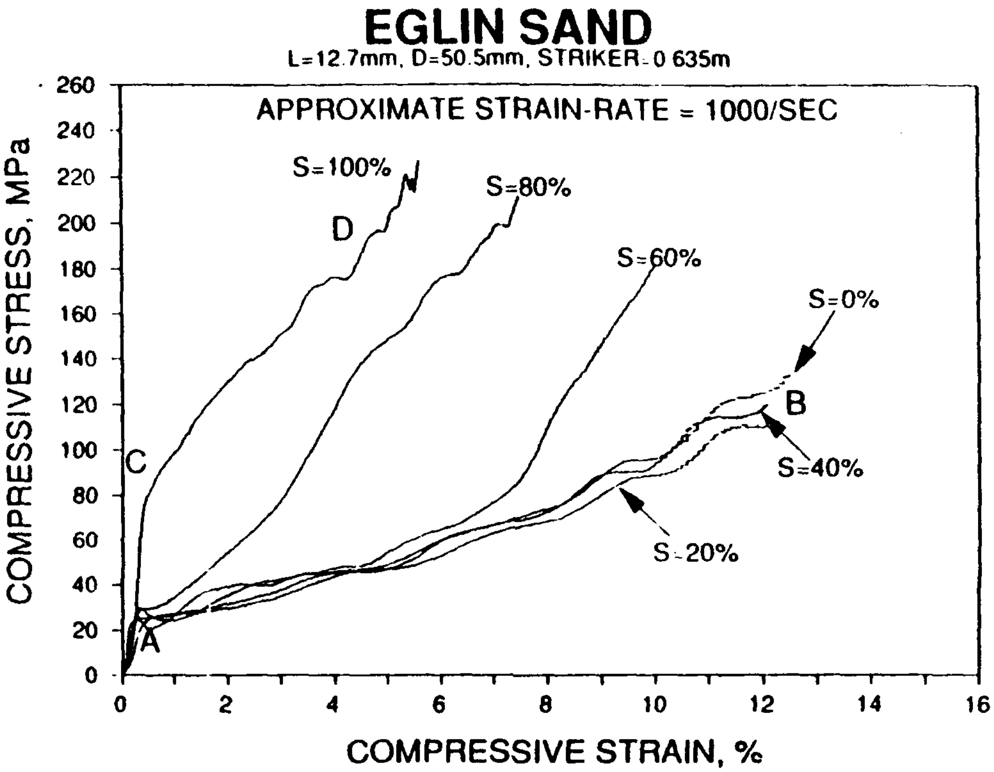

FIGURE 8a. SHPB Uniaxial Undrained Compressive Stress-Strain Response for Eglin Sand as a Function of Saturation (Strain Rate is Approximately 1000/sec).

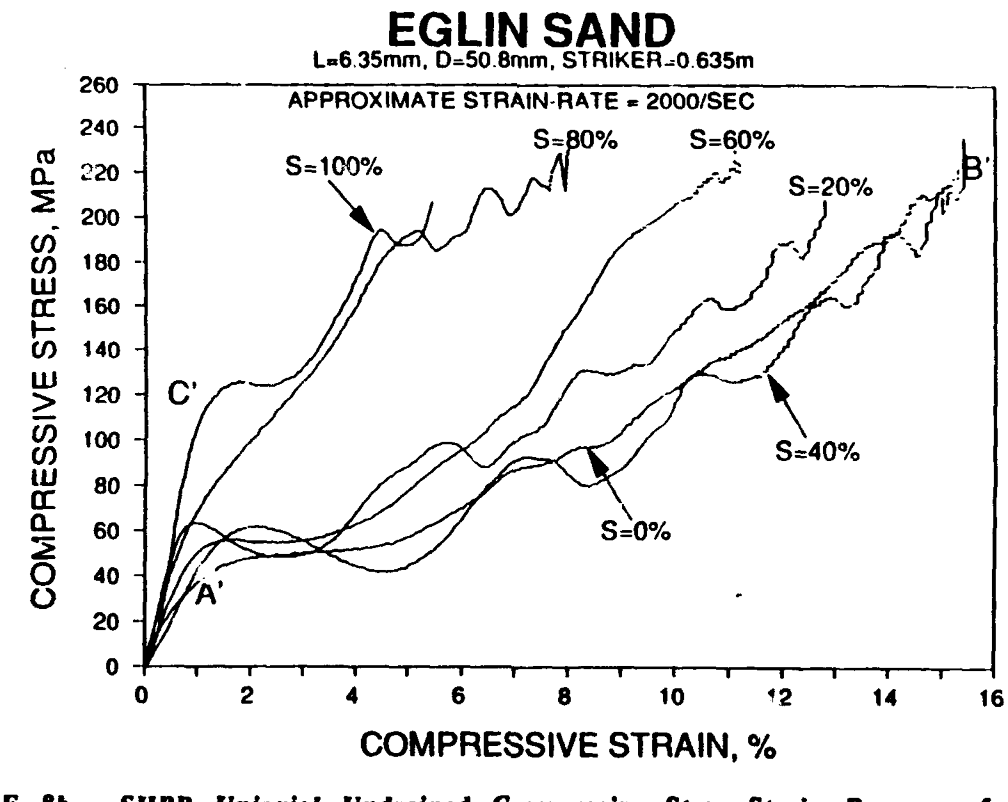

FIGURE 8b. SHPB Uniaxial Undrained Compressive Stress-Strain Response for Eglin Sand as a Function of Saturation (Strain Rate is Approximately 2000/sec).

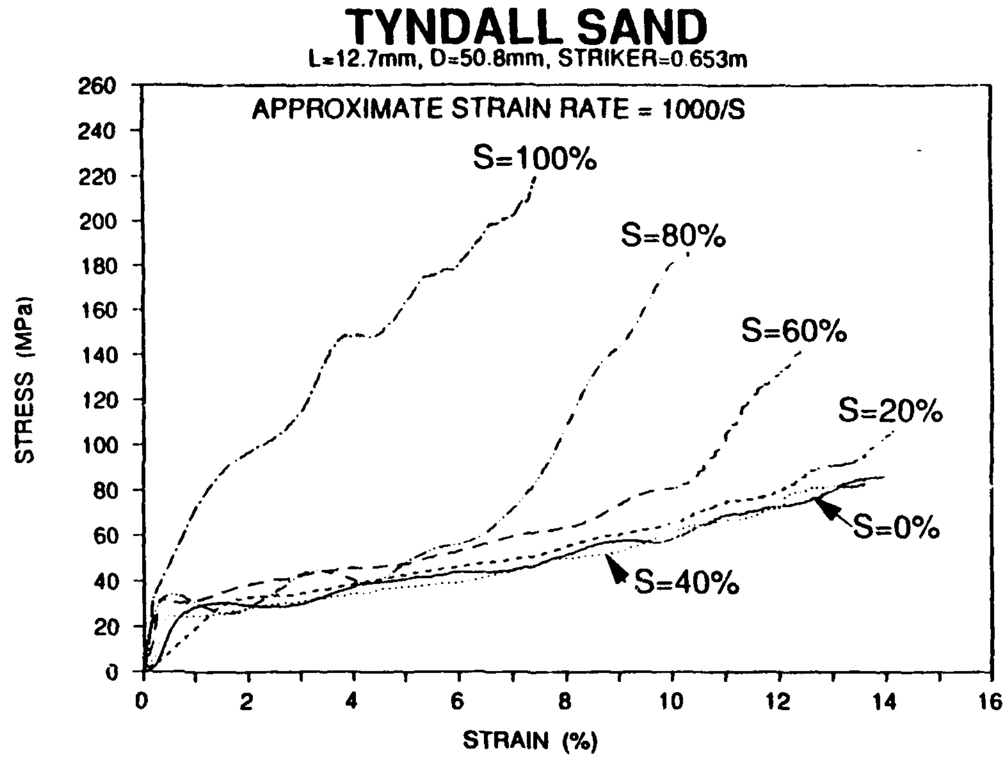

FIGURE 9. SHPB Uniaxial Undrained Compressive Stress-Strain Response for Tyndall Sand as a Function of Saturation (Strain Rate is Approximately 1000/sec).

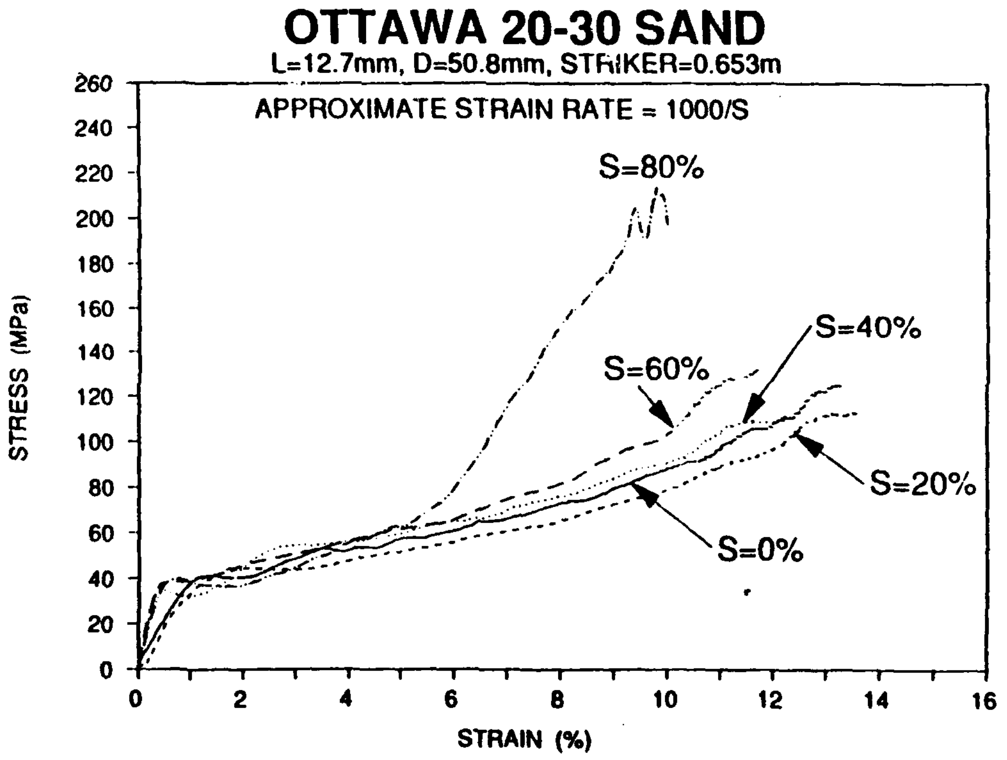

FIGURE 10a. SHPB Uniaxial Undrained Compressive Stress-Strain Response for Ottawa 20-30 Sand as a Function of Saturation (Strain Rate is Approximately 1000/sec).

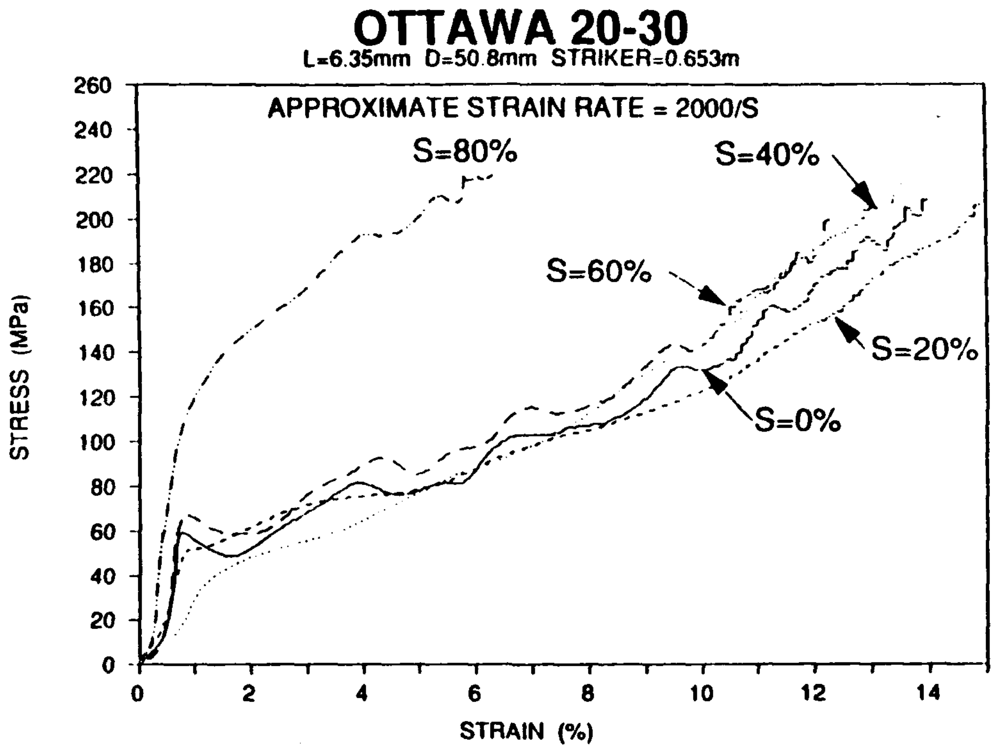

FIGURE 10b. SHPB Uniaxial Undrained Compressive Stress-Strain Response for Ottawa 20-30 Sand as a Function of Saturation (Strain Rate is Approximately 2000/sec).

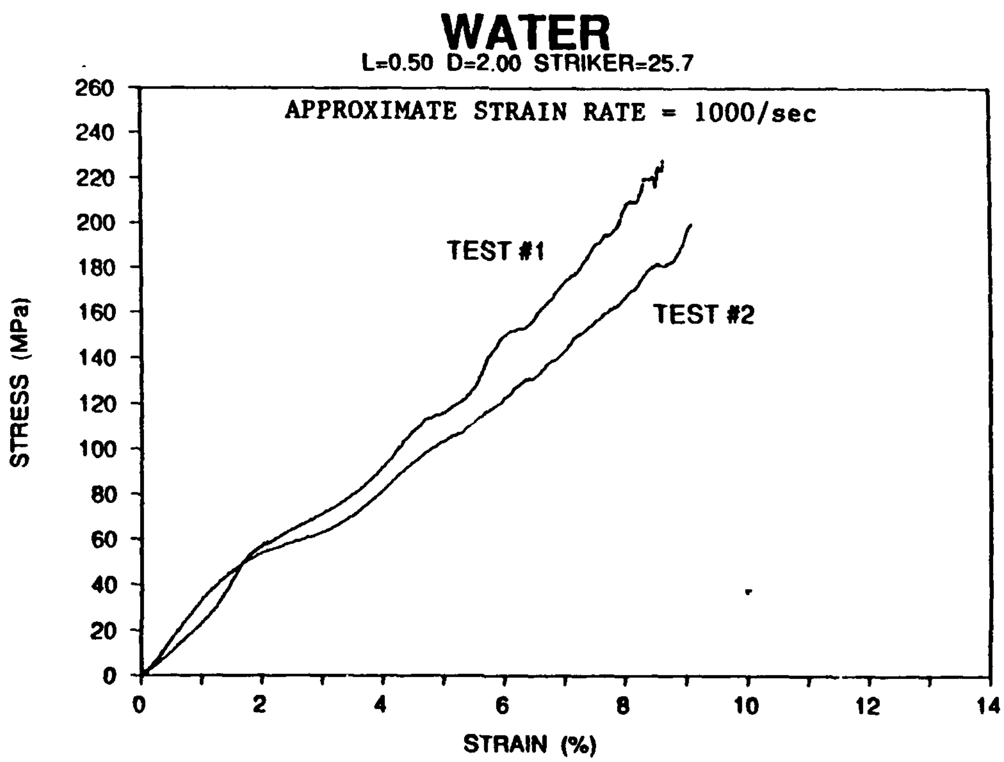

FIGURE 11. SHPB Uniaxial Undrained Compressive Stress-Strain Response for Water (Strain Rate is Approximately 1000/sec).

Differences in the basic features between the curves when comparing the three different soils are most likely due to differences in grain size characteristics (i.e., size and distribution).

The term "lock-up" as used herein refers to the sharp increase in the slope indicating a stiffening behavior of the stress-strain curve at some compressive strain (after the initial loading portion of the curve). Since the slope is approximately that obtained for SHPB tests conducted on pure water (the saturated soil-water mixture slopes are slightly greater than that for water due to density differences), the lock-up strain represents a condition of full saturation in the soil due a reduction in void space. Therefore, the results suggest that the stress-strain response is dominated by the water phase (saturated soil-water mixture) from the initiation of the lock-up strain and beyond, while the soil skeleton (unsaturated soil-water mixture or soil-air mixture if dry) dominates the response from the start of loading up to the lock-up strain. Lock-up was not developed at lower saturations even at large strains, which indicates that insufficient pore space reduction occurred (i.e., not enough compressive strain was developed). Therefore, larger amplitude stress loadings (higher compressive strains) with longer pulse lengths need to be applied so that the stress-strain response at the lower saturations can be determined.

Table 2 shows a comparison of the approximate measured lock-up strains estimated from the SHPB data (Figures 8, 9 and 10) and the theoretical compressive strain required to reach full saturation (i.e., lock-up). The theoretical calculations only account for the amount of air-filled void space in each soil based on initial void space and initial saturation. The results generally do not agree with the measured compressive strains at lock-up. However, the interaction of the various components of the soil-water mixtures are not accounted for in these calculations. Since the soil-water mixture represents a multi-phase material, a complete description of the problem becomes very complex since in general there are four different interacting components contributing to the bulk response: a) the soil skeleton; b) the pore air; c) the pore water; and d) the individual grain stiffnesses. While the individual stress-strain response of each component can be determined separately, it is their interrelationship that determines the overall behavior. A further complicating factor is that the dominance of any one (or combination) of these components changes depending upon the initial conditions and those during loading (i.e., strain magnitude). However, these interrelationship are not well defined or understood, particularly for transient dynamic loading conditions.

Even though this investigation is not exhaustive in extent, it does shed some new light (and poses new questions) on the undrained dynamic behavior of unsaturated soils at high strain rates and represents an important first step towards establishing an understanding of this phenomenon. In addition, this research has also demonstrated that the SHPB system is a viable technique for high strain rate dynamic geotechnical testing of unsaturated, saturated and dry soils, and provides a framework for conducting further studies using the SHPB with soils. While the saturation dependent uniaxial stress-strain behavior observed in this study has been theorized and hypothesized in the past by other researchers, these results appear to be the first detailed measurements of this phenomenon for undrained uniaxial confined compressive loadings at high strain rates.

TABLE 2. Comparison of Approximate Measured Compressive Strains at Lock-Up and Theoretical Compressive Strains to Reach S=100% Based on Initial Void Ratio and Initial Saturation.

	EGLIN[a]			TYNDALL[b]			OTTAWA 20-30[c]		
S (%)	ε^d	ε^e	ε^f	ε^d	ε^e	ε^f	ε^d	ε^e	ε^f
0	33.8	(g)	(g)	39.5	(g)	(i)	35.3	(g)	(g)
20	27.0	(g)	(g)	31.6	(g)	(i)	28.2	(g)	(g)
40	20.3	(g)	(g)	26.7	(g)	(i)	21.2	(g)	(g)
60	13.5	7.5	6.0	15.8	10.5	(i)	14.1	(g)	(g)
80	6.8	3.0	2.0	7.9	6.5	(i)	7.0	6.0	1.5
100	0.0	0.5	1.0	0.0	0.5	(i)	0.0	? (h)	? (h)

Note:
- a Eglin sand at initial void ratio = 0.510
- b Tyndall sand at initial void ratio = 0.654
- c Ottawa 20-30 sand at initial void ratio = 0.545
- d Theoretical compressive strain required to obtain S=100% based on initial void space and saturation. For S=0%, this represents a condition of zero air voids
- e Approximate lock-up compressive strain from SHPB data for strain rate = 1000/sec.
- f Approximate lock-up compressive strain from SHPB data for strain rate = 2000/sec.
- g Unable to obtain lock-up strains using the 0.653 m (25.7 inch) projectile at 690 kPa (100 psi).
- h Unable to obtain reliable data for Ottawa 20-30 sand at S=100%.
- i Data not obtained for Tyndall sand at this strain rate.

SECTION IV

RECOMMENDATIONS

The understanding of load transfer mechanisms in unsaturated soils is very limited at present. Further investigations are necessary and will provide important information to the U.S. Air Force with respect to military protective construction and survivability designs. Recommendations for further research include:

1) The fundamental aspects of load transfer mechanisms, material phase interactions and the effects of boundary conditions in compacted unsaturated soils should be examined for both static and dynamic loading conditions in a comprehensive experimental testing program. Such a program should include the following:

a) A more detailed investigation should be undertaken to study strain rate effects in unsaturated soils using the SHPB. In addition, centrifuge tests using scaled explosive charges should also be conducted. Based on the results of such studies, data will be available for numerical and theoretical model development and a better fundamental understanding of the phenomenon can be obtained.

b) Numerical experiments should be conducted to model the behavior of dynamically loaded unsaturated soils using techniques such as the Distinct Element Method (DEM) in which load transfer mechanisms and material phase interactions can be examined. The modeling effort would be guided by the experimentally derived stress-strain data from SHPB and centrifuge tests.

2) A series of fully instrumented, carefully controlled small scale field explosive tests should be conducted in close coordination with additional laboratory studies. Test parameters should include variations in saturation, compaction methods, boundary conditions and applied energy. Field instrumentation should provide measurements of input energy, soil deformation

and stress and transmitted energy as a minimum. Results will be useful for relating laboratory and field measured soil behavior, providing valuable input for material models.

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